Liquefaction Risk Assessment for the State Highway 2: Dowse to Petone Upgrade Project.

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The site of the proposed realignment of State Highway 2 from Petone overbridge through to the Dowse intersection, Hutt Valley, New Zealand is in a region of high seismicity. The alignment is subject to very large seismic design loadings (1000 year Return Period PGA of 0.65g, Maximum Credible Earthquake PGA 1.0g) due to its proximity to the active Wellington Fault (M_w 7.4). Following a site-specific seismic hazard assessment, the geotechnical assessment of the site included a liquefaction analysis addressing likely magnitude of induced ground subsidence and effects on bridge piles, MSE abutments/ ramps, and nearby commercial properties at Dowse. Results of the seismic assessment present significant challenges to bridge and MSE wall designers for this project in achieving compliance with Transit New Zealand design philosophy requirements. This paper discusses methodologies adopted for the liquefaction assessment, results, and implications for the design of the structures on the project.

INTRODUCTION

As part of Transit New Zealand's highway network in the Wellington region, State Highway 2 is a strategic corridor linking Wellington city, in the south of the North Island, with the Hutt Valley suburbs, where it continues over the Rimutaka Hills into the Wiararapa and northwards. The SH2 Dowse to Petone project extends 2.7 km from the Petone foreshore northwest in line with the Western Hills, through two major intersections at Korokoro and Dowse (Figure 1). As part of the projected realignment of this road, intersections at Korokoro and Dowse are to be replaced with interchanges allowing traffic to continue along the state highway unimpeded. To achieve this, a number of overbridges and connecting Mechanically Stabilised Earth (MSE) ramp structures are to be constructed to link the existing side roads to both the highway, and the nearby Hutt Road –a major local road that lies adjacent to SH2.

The significance of the proximity of the Wellington Fault, and the geological setting overall, was evident from the results of the site specific seismic hazard assessment carried out by Beca in 2003, indicating that regional seismicity was heavily dominated by the close proximity of this fault, and the other regionally significant earthquake sources such as the West Wairarapa Fault, which produced New Zealand's largest magnitude earthquake in historical times (M_w 8.5, in 1855).

The recent geological history of the site has included periods of fluvial deposition from periodic avulsion of the Hutt River, in addition to periods of marine deposition in response to absolute sea level changes. Although some of these deposits have densities that fall within the range specified in the literature as being "liquefiable" most are of medium dense consistency and have some silt content, placing them outside the bounds of what the literature has *traditionally* focussed on in terms of what is known to be "liquefiable" (i.e. loose, uniformly graded (very well sorted), medium grain sized sands). This is because of the high levels of acceleration anticipated.

How liquefaction may affect nearby engineering structures can be assessed from observation of these phenomena in past seismic events; sand boils, large scale ground settlements, loss of bearing capacity resulting in foundation failure, lateral spreading resulting in shoving/ sliding towards an unsupported margin, buoyant displacement of buried services causing disruption. This paper provides a brief appraisal of the liquefaction risk assessment process as it was applied to the project, and discusses some of the outcomes of the study.

THE GEOLOGICAL SETTING AND PROFILE

The geological setting of the Hutt Valley region is dominated by active tectonics associated with the Wellington Fault, which runs from Cook Strait, through the Karori Reservoir, Thorndon, along the northwest side of the harbour and the west side of the Hutt Valley. The Western Hills are the remnants of a degraded fault scarp, progressively uplifted and eroded along the length of the Wellington Fault (Figure 1). The rock that forms the Western Hills is of the Torlesse Complex, and is commonly interbedded greywacke sandstone and argillite siltstone. As a legacy of its complex history, these rocks have undergone extensive deformation associated with periods of tectonic extension and compression, involving warping, tilting, folding and faulting of the rock mass, leaving it extensively fractured and jointed, and locally described as a "fault breccia".

The fault scarp extends some 300 m below sea level adjacent to the shoreline at Petone (Begg and Mazengarb, (1)). The fault is projected steeply through Quaternary sediments to a surface expression some 250 m and 150 m east of the Western Hills, at Korokoro and Dowse interchanges respectively.

The fault rupture, estimated to have been caused in the past by a M_w 7.4 level earthquake, is such that the eastern wall of the fault moved vertically down 0.5-1 m, and horizontally to the south 4.0 m relative to the western wall.

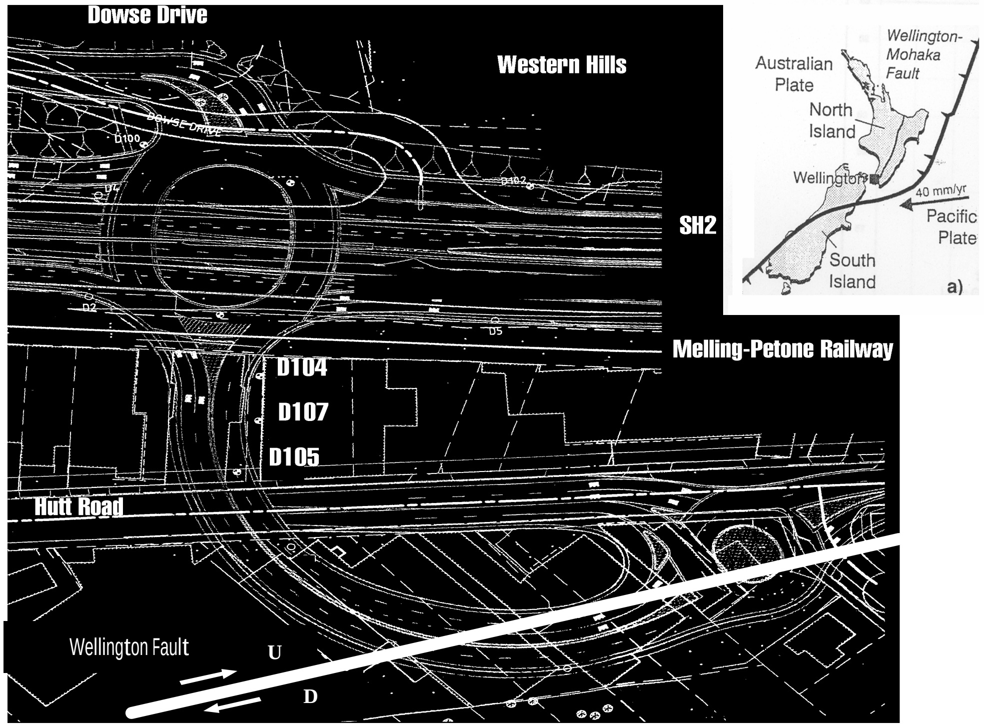

Figure 1: Location Plan for SH2 Dowse Interchange and connecting overbridge to Hutt Road. (1a) Inset: Tectonic plate boundary in relation to Wellington.

The location of the fault at the Dowse-Hutt Road site has been investigated by trenching (IGNS (2)). Findings from this study and mapping of surface expressions in the vicinity enable a confident positioning of the primary fault rupture zone to the east of Hutt Road, and probably crossing the proposed MSE ramp structure at Hutt Road.

The Hutt Valley floodplain is a tectonically controlled sedimentary basin, infilled with glacially derived alluvial gravels of greywacke origin, interglacial marine silts, and sands associated with sea level changes. This has resulted in distinct layers of fine-grained and coarse-grained materials. Near shore marine and beach sediments inter-fingering with fluvial and fan gravels deposited by the Hutt River and minor tributaries coming off the Western Hills, form the most recent deposits in the project area. Colluvium and rockslide debris occur to the west of the existing highway, and manmade fills surround and extend beneath the existing highway, most notably at Korokoro (5-6 m thickness).

GEOTECHNICAL CONDITIONS

The focus of the geotechnical investigations carried out for detailed design was on determining the founding conditions for the bridge piles throughout the project, and for the MSE ramp structures. Due to the soils on the project being largely medium dense greywacke-derived alluvial and colluvial soils, boreholes were favoured over CPTs.

During the preliminary phase investigations (1998), medium-dense marine sands were identified to the east of the Dowse Interchange in one borehole; however no laboratory testing was carried out, leaving the possibility of liquefaction as a design issue speculatory and inconclusive. Thus during the detailed design phase investigations, additional boreholes were used to attempt to determine how extensive any potentially liquefiable materials were, and understand the likely impact this phenomenon would have on the proposed structures.

A summary of the vertical sequence of materials at Dowse to the east of SH2, towards Hutt Road, is presented in Table 1 (Beca, (3)).

Table 1
Dowse (East of SH2) Summary of Materials

Approx Depths (m)	**Material Description**	**Corrected SPT *N* values ($N_{1(60)}$)**	**Comments**
0-2	Silty Sands with some gravel (FILL/ disturbed natural ground)	20-50 *(Typ. 37)*	Varies in thickness and composition.
2-4	Gravelly Sands and Silts (ALLUVIUM)	22-50 *(Typ. 42)*	Largely sandy gravels extend from the highway east. Approx. 3 m thick.
4-18	Silty Sands (MARINE to MARGINAL MARINE/ ALLUVIUM	13-50 *(Typ. 20)*	Sands with varying silt contents. Uniformly graded sands extend from immediately east of SH2, to east of Hutt Road.
18-?	Silty Sandy Gravels (ALLUVIUM)	50	Alluvial Gravels identified from east of Railway eastwards to eastern side of the Wellington fault.
-	Greywacke/Argillite basement rock	50+	Not encountered east of the Railway.

The sands identified between 4 and 10 m depth, are described as medium dense grey uniformly graded fine sand, occasional gravels and shells. SPT $N_{1(60)}$ range for this deposit: 13 -25.

Correlations of the soil deposits encountered during the geotechnical investigations showed that the bedrock dips steeply away from the Western Hills, such that it is not encountered in boreholes to the east of SH2. Marine sediments were only encountered to significant depths (13.5 m east of Hutt Road on the downthrown side of the fault), on the eastern side of SH2, indicating the on-lapping of the marine incursion across the site from east to west. Boreholes to the east of the Wellington fault were found to have the same marine deposits as identified west of the fault, however at these locations SPT testing recorded much greater densities (SPT N>30). This difference may be in part to lower energy deposition where the marine deposits onlap, or is perhaps a function of directional earthquake energy effects.

LIQUEFACTION ASSESSMENT METHODOLOGY.

Site Seismicity and Liquefaction Assessment Philosophy

The liquefaction assessment methodology was adopted to meet the structural Design Objectives (Beca, (4)):

Objective 1: Withstand major earthquake shaking (assumed to be that which has a 10% probability of exceedence in the 100 year design life of the bridge structures in this project (i.e. approximately 1000 years) with a reasonable margin against collapse, and;

Objective 2: Withstand the most severe earthquake shaking (assumed to be equivalent to that from the MCE with a median plus one standard deviation predicted shaking) with a low margin against collapse.

For earth structures, including MSE with a period at or close to zero, the 1000-year return period (design base earthquake) Peak Ground Acceleration (PGA) is 0.65 g. For MCE level of shaking, the recommended PGA for zero period structures is 1.0 g.

On finding evidence in the geotechnical profile for potentially liquefiable marine sands, a liquefaction assessment was carried out, to verify the layers would indeed liquefy under the design base earthquake, and attempt to quantify the likely effects on the proposed structures for the Dowse-Hutt Road sites.

Source Information and Parameters Used

The site-specific hazard report included graphs of recurrence of PGA for various magnitude ranges (Figure 2) and the response spectra for the MCE (Wellington Fault), which were used to evaluate the likelihood of liquefaction occurring.

Soil grading information for individual soil layers was based on nearest available grading test data in the borehole column. Some samples obtained from open nose SPT tests during drilling had to be combined to provide sufficient sample for testing, reducing the accuracy of the sieve grading test, but providing a ballpark figure for fines content. Hydrometer testing provided grading of the silt and clay portion of the samples, an important consideration for the liquefaction assessment of sands with some fines present.

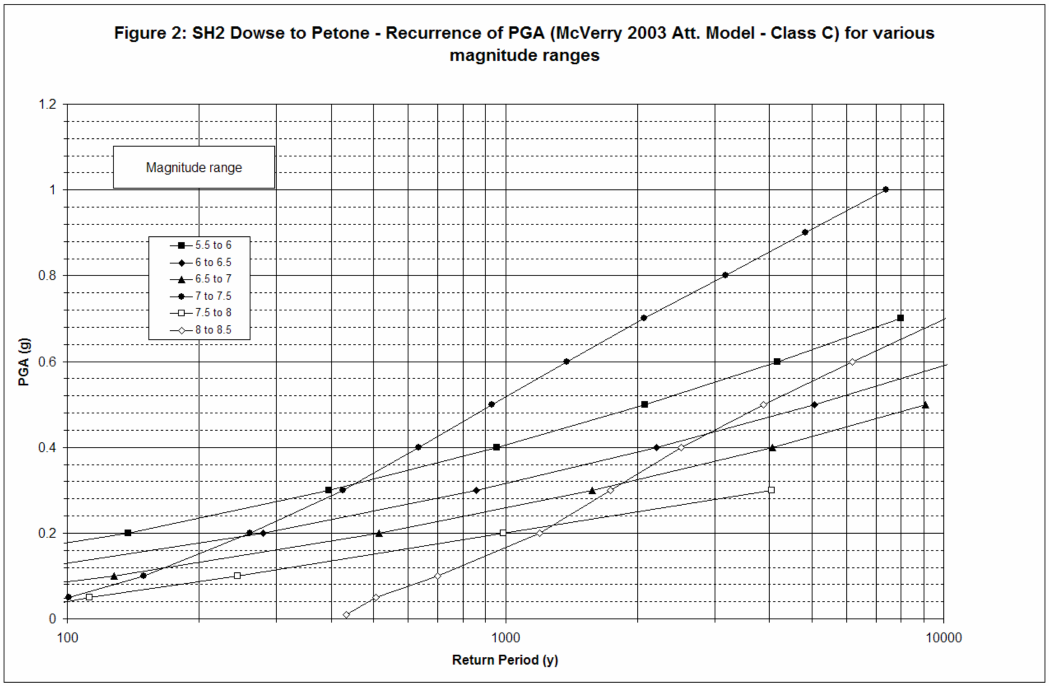

Figure 2: Recurrence of PGA for various magnitude ranges. Magnitude ranges 7-7.5 and 8-8.5 have much larger increases in PGA with return period, due to the weighted contribution provided by Wellington and West Wairarapa faults respectively. *Source: Beca (4)*.

Assessment Method followed

There are many factors that affect whether a certain soil at a certain site, under a certain magnitude earthquake will liquefy. As part of the assessment, all of these factors need to be considered (Kramer (5)):

Historical Criteria

Evidence of past liquefaction at a site may well be evidence of susceptibility of soils to further liquefaction. There are no known reports of liquefaction damage to this site occurring during the M_w 8.5 Wairarapa earthquake of 1855, however much of the Lower Hutt Valley was an undrained wetland at the time, and any evidence of liquefaction may have been missed. Historical earthquakes also lessen the risk of further occurrences of liquefaction under the following criteria:

Geological Age and Origin

It is generally considered that older deposits have a lower risk of liquefying than younger deposits, as older deposits are more likely to have experienced similar seismic events, already resulting in densification of the soil. Weathering effects over time may lead to a cement-like bonding between grains, such that there is a higher amount of energy required to break the initial particle configuration in order to attempt to densify the deposit. At Dowse-Hutt Road, the proximity of the marine sand to the surface deposits, and our understanding of the geological history of the near surface deposits, suggest this marine sand was laid down in the Holocene (<6Ma), prior to shoreline regression to its current location at Petone. It may have experienced anywhere between 1 and 10 prior M_w 7.4+ prehistoric seismic events.

Particle Size and Classification

In general, the potential for a soil to liquefy decreases with increasing fines content in particular when those fines are plate-like minerals with high surface area such as clays. For this reason, Plasticity Index, liquid limit, and clay content are considered to be good test criteria for whether a particular soil will exhibit liquefaction or not. Non-plastic silts are known to liquefy. Using Tsuchida and Hiyashi (6) curves of liquefaction susceptibility based on grain size, some sands, over the depth range typically 5-7 m at Dowse-Hutt Road with non plastic silt fines in the range 13 - 43 % fall into the “easily” to “very easily” liquefy categories. (Figure 3).

Density

Beyond a certain density, the earthquake energy will be transferred sufficiently through the soil via direct grain-to-grain contact that the granular configuration of the sand will not attempt to recompact, thereby avoiding placing stress on water within the pore spaces. Also, beyond a certain depth (say a nominal 10 m), liquefaction will be highly unlikely to occur due to the additional vertical stress on the soil particles providing resistance to the effort to recompact the soil when shaken. In liquefaction assessment procedures adopted in the literature, SPT and CPT are the common tools for determining the relative density of the soil. The NCEER (Youd and Idriss (7)) method suggests that soil in the “Dense” category, (SPT *N* = 30+) has an extremely low risk of liquefaction, regardless of the Cyclic Stress Ratio that may develop.

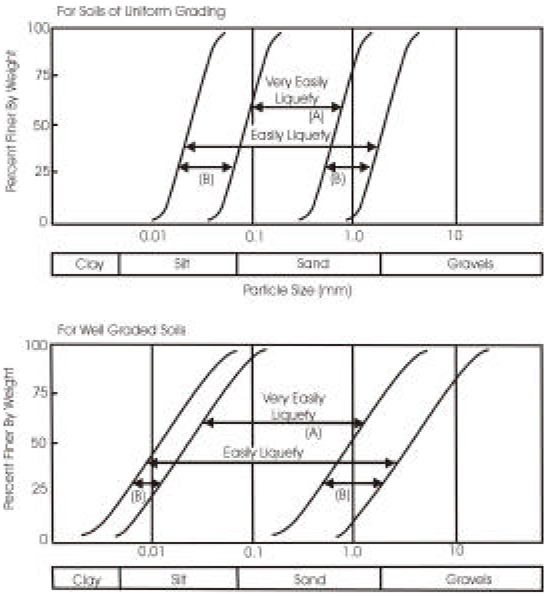

Figure 3: Tsuchida and Hiyashi (6) Grain size distribution curves for liquefaction susceptibility.

The marine sands identified as being potentially liquefiable when plotted on the Tsuchida curves, fall within the Very-easily to easily liquefy categories for uniformly graded soils.

Calculation of Resistance to Cyclic Stress

Under Objective 1, the susceptibility of the soils to liquefaction was assessed using the probabilistic method outlined in the paper by O’Brien et al (8). The calculations adopted for assessing liquefaction were taken from the NCEER report. Corrected SPT data was used from boreholes at locations adjacent to the proposed structures at Dowse and Hutt Road sites. Magnitude Scaling Factors (MSF) were applied as per NCEER. The upper bound values of Andrus and Stokoe and lower bound values of Idriss were averaged for our calculations, and interpolated for the midpoint of ranges corresponding to those magnitudes provided in the site specific hazard report (Figure 2).

The probabilistic method is summarised as follows:

1. The soil stratum below the water table was subdivided into ‘sub layers’ of 0.5 m thickness.
2. PGA likely to cause liquefaction for various magnitude ranges was assessed for each sub layer. Earthquakes of magnitude less than 5.5 were excluded, as they are unlikely to cause liquefaction due to the short duration of shaking caused by these events. However, if low magnitude events were included the result would not change significantly, as the frequency of liquefaction from these events would be very low. This was calculated through solving to obtain a factor of safety (FOS) of 1.0 with corresponding MSF’s applied.
3. The return period associated with the PGA to cause liquefaction for a particular magnitude was assessed from Figure 2.
4. The Expected Annual Frequency was calculated by inverting the return period.
5. The return period for liquefaction for a particular layer is the inverse of the sum of the expected annual frequencies for each magnitude level.

The susceptibility of a soil layer to liquefy under Objective 2 was assessed by taking the PGA recommended by the site-specific hazard report, as being the spectral acceleration at a period of zero for the MCE. The liquefaction susceptibility in this case was assessed non-probabilistically as it is a definite single event case that was checked. This involved a calculation of the cyclic stress ratio (CSR) using the MCE PGA, to determine directly whether liquefaction occurred given the cyclic resistance ratio (CRR) determined by the properties of the soil. The PGA at which liquefaction was initiated for these deposits was also determined.

RESULTS OF LIQUEFACTION ASSESSMENT.

At Dowse interchange, the results of this assessment quantified the risk of liquefaction in terms of the expected return period for liquefaction to occur. Results from Boreholes D104, D105 and D107 (Refer Figure 1) demonstrated the susceptibility of the marine sand deposit (5-7m depth) to liquefaction. Despite the fact that at their location (between the railway and Hutt Road) the marine sand deposit is only in the order of ~ 2 m thick, this deposit (with similar and lower SPT readings) is observed in historical boreholes on the eastern side of Hutt Road at up to 6.5 m thick. This indicates that potentially liquefiable soils cover a broad area, affecting the abutment ramps, and the piles of the proposed Hutt Road overbridge. These historical borelogs were not assessed beyond inspection as no grading tests were performed on samples preventing an accurate analysis.

Return periods for the occurrence of liquefaction in soil layers between 5 m and 7.5 m depth in boreholes D104, D105 and D107 average 335 years, with a minimum of 205 years, and a maximum of 680 years. A loose to medium dense silty sand at 9-10m depth in D105 had a liquefaction return period of only 120 years using this method.

Level ground settlements were estimated using the method of Ishihara and Yoshimine (9), and calculated to be in the order of 100-200 mm west of Hutt Road, with settlements to the east of Hutt Road potentially up to three times this amount. This ground settlement is expected to induce a downward force due to negative skin friction on the piles supporting the bridge piers and abutments. The load was assessed from the thickness of soil above the water table that would form a crust in the event of liquefaction. Lateral resistance provided by the liquefiable deposit was ignored under the seismic load case, as per the requirements of the Transit NZ Bridge Manual (10), which conforms with the findings of Ishihara (11).

Lateral spreading was considered as a possibility, however the nearest unsupported margin is the Hutt River over 1 km away, and borehole logs east of the Wellington fault show deposits too dense to liquefy, proving a restraint to lateral spreading.

CONCLUDING REMARKS

The SH2 Dowse to Petone project has provided a challenge in ascertaining the seismic affects on the project, complicated by the presence of the Wellington Fault transecting the site. In addition to the anticipated large fault movements, and high design seismic accelerations, liquefaction was found to be a risk that required special design consideration for the structures on the project.

As part of the assessment of liquefaction risk, a clear understanding of the site geology and how it affects the likelihood and extent of potential liquefaction was required, followed by an understanding of the impacts that liquefaction is likely to have on the design of the proposed structures, in terms of additional loads on piles, affects on MSE footings, level ground settlements etc. The use of probabilistic methods to evaluate liquefaction risk is considered important to maintain the integrity of the nature of the seismic hazard assessment data (being probabilistic).

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